HYDRAULICS OF BRIDGES

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Joint
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Informational Report

HYDRAULICS OF BRIDGES

PROGRESS REPORT OF THE TASK FORCE ON HYDRAULICS OF BRIDGES OF THE HYDRAULIC STRUCTURES COMMITTEE HYDRAULICS DIVISION, AMERICAN SOCIETY OF CIVIL ENGINEERS

TO: K. B. Woods, Director
Joint Highway Research Project

File: 9-8-2

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

September 18, 1964

Attached is an informational copy of a Progress Report of the ASCE Task Force on Hydraulics of Bridges. A summary of this report was presented by Professor J. W. Delleur, Chairman of the Task Force, at the Vicksburg, Mississippi meeting of the Hydraulics Division on August 19, 1964, as part of a technical session on Hydraulics of Bridges. Professor Delleur became a member of the Task Force on Hydraulics of Bridges as a result of his work on "Hydraulics of Bridges" co-sponsored by the Project, the Highway Commission and by the BPR.

Professor Delleur would appreciate receiving comments and suggestions from the members of the Project staff and from the Engineers of the Indiana State Highway Commission.

The paper is presented to the Board as information.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Secretary

HLM:bc

Attachment

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Informational Report

HYDRAULICS OF BRIDGES

Progress Report of the Task Force
on
Hydraulics of Bridges
of the
Hydraulic Structures Committee
of the
Hydraulics Division
American Society of Civil Engineers

Draft
Not for Publication

Presented at the ASCE Hydraulics Division Meeting
Vicksburg, Mississippi, August 19, 1964

PREFACE

The Task Force on Hydraulics of Bridges was created in January 1960 operating under the Committee on Hydraulic Structures. The original members were: the late H. K. Liu, Messrs. J. W. Delleur, J. B. Herbich, and H. J. Tracy, Chairman. Mr. D. E. Schneible was appointed to replace Mr. Liu in April 1961. J. W. Delleur was appointed Chairman in November 1962, and E. M. Laursen was added to the Task Force in October 1963.

The general objectives of the Task Force were stated by A. J. Peterka, then chairman of the Committee on Hydraulic Structures as follows:

"To stimulate the accumulation and dissemination of information regarding design methods and performance characteristics of the Hydraulics of Bridges; to foster the prompt utilization of research results in the development of improved design procedures."

The Task Force has attempted to meet these objectives by preparing an comprehensive outline to delineate in a systematic fashion the problems involved in the hydraulic design of bridges, and to present an annotated bibliography summarizing the state of the art. This outline attempts to be more than a statement of the available knowledge, it should be a thoughtful discussion of the problems that have arisen and yet remain, together with some recommendations and suggestions for research. The following is a draft of this outline.

The task Force has also attempted to disseminate information regarding

Hydraulics of Bridges by organizing technical sessions on this subject. This is the second technical session organized by the Task Force.

The first program organized by the Task Force was at the Boston meeting in May 1960. Papers were presented by

J. B. Herbich on "The Effect of Spur Dikes on Flood Flows through Bridge Constrictions:

P. F. Biery and J. W. Delleur on "Hydraulics of River Flow under Arch Bridges"

J. Shen on "Flow through Multi-Opening Constrictions"

W. W. Sayre and M. L. Albertson on "The Effect of Roughness in Rigid Open Channels"

HYDRAULICS OF BRIDGES

1. INTRODUCTION

The various aspects of bridge design, such as location, alignment and the structural foundation, hydrologic and hydraulic design are closely inter-related by the overall considerations of service, safety and economy. The best solution of a problem in one of these phases may involve appreciable modification of the initial proposals of one or more of the other phases. For example, excessive velocities through a bridge waterway may require a lengthening of the span or even a relocation. Again, it may be practicable, from the hydraulic design aspect, to vary the height-to-span ratio of a bridge over an appreciable range and the best ratio being fixed by structural and economic consideration.

This inter-relationship is to be borne in mind in the following outline of the elements of hydraulic design. First, there are listed the design data which may be required by the hydraulic designer, together with brief indications of certain matters requiring investigation, discussion or standardization. Next, the aspects of hydraulic design are presented, with comments on the methods in current use. Finally, there is set out a list of topics based upon which a comprehensive program of study might be undertaken.

2. HYDRAULIC DESIGN DATA

2.1 SITE USE

At some sites, the hydraulics of a proposed bridge may be superseded by other factors, such as topography, navigation, recreation, fish-and-wildlife, geology.

2.1.1 SITE IS AT A GORGE

The gorge will contain the predictable flows of the stream. The problem of a structure spanning the gorge is paramount to the hydraulics of the bridge.

2.1.2 SITE IS A NAVIGABLE WATERWAY

The navigation considerations will require a bridge of such height and length which exceed the requirements for the hydraulics of the bridge.

2.1.3 SITE IS IN A RECREATION AREA

The passage of boats may require a structure larger than needed for hydraulic reasons.

2.1.4 SITE IS IN A FISH OR WILDLIFE HABITAT

A habitat for fish or wildlife may be damaged or destroyed by the construction of earth embankment. A bridge may be constructed in lieu of the embankment and, thus, the hydraulics are a minor consideration.

2.1.5 SITE HAS UNSUITABLE SOILS

Soils which are not capable of supporting an embankment must be replaced or bridged. If the economics indicate a bridge, the hydraulics of the structure may be insignificant.

2.2 HYDROLOGY

The determination of the required waterway area of a bridge, the selection of the size of a culvert, or the design of a highway drainage system in general require an accurate estimate of the peak discharge that is expected to pass through the structure. Ven Te Chow has published a general review of Hydrologic Studies of floods in the United States.(0)

TABLE 1

Major organizations in the United States engaging in hydrologic studies of floods and their main interests related to the studies.
(after Chow[0])

A. Federal Agencies
   (a) Department of Agriculture
      a. Soil Conservation Service (1935) - In upstream flood control and agricultural watershed management related to minor floods.
      b. Agricultural Research Service (1954) - In basic research to obtain information needed for upstream flood control and watershed management.
      c. Forest Service (1906) - In flood survey and effect in forest areas.
   (b) Department of the Army
      a. Corps of Engineers (1802) - In flood control for major floods.
         1. Office of the Chief of Engineers and offices of 10 Division Engineers - In general and particular problems in the District.
         2. Mississippi River Commission (1893) - In Mississippi River flood problems.
         3. Waterways Experiment Station (1928) - In model studies.
   (c) Department of Commerce
      a. Weather Bureau (1940) - In flood data collection and forecasting and determination of design criteria.
      b. Bureau of Public Roads (1949) - In minor and medium floods related to the design of bridges and culverts.
   (d) Department of the Interior
      a. Bureau of Reclamation (1902) - In flood problems for reclamation projects.
      b. Geological Survey (1879) - In flood data collection, survey, and analysis.

B. Federal Corporation
   (a) Tennessee Valley Authority (1933) - In Tennessee Valley flood Problems.

C. Public Corporations - In flood problems of the particular region; for examples:
   (a) Pittsburgh Flood Commission (1908)
   (b) Miami Conservancy District (1914)
   (c) Los Angeles County Flood Control District (1915)
   (d) Franklin County Conservancy District (1915)
   (e) Muskingum Valley Conservancy District (1933)

D. State and Local Agencies.

E. Educational Institutions - Public and private universities and colleges.

F. Technical Societies - For examples:
   (a) American Society of Civil Engineers (1852)
   (b) American Geophysical Union (1919)
   (c) Society of American Military Engineers (1919)
   (d) American Society of Agricultural Engineers (1920)

2.2.1 FLOWS AND STAGES IN THE UNCONSTRICTED STREAM.

The flows and stages vary from a minimum to a maximum. The knowledge of the maximum design stage and the corresponding flow are usually required for the hydraulic design of bridge openings. The Daily River Stages at Gage Stations on the Principal Rivers of the United States have been compiled between 1856 and 1948 by the Signal Service, United States Army, and since 1948 by the Weather Bureau.[1]

By means of stage-discharge curves, which are generally available from the USGS regional offices for the principal gaging stations, the stage information may be transformed into a discharge information. In addition there is a number of publications which deal with regional flood records, and are published by the Weather Bureau, the USGS and state agencies.[2]

2.2.2 DESIGN FLOOD - BRIDGE ON GAGED STREAM

If the bridge is at or near a gaging station for which records are available, the largest discharge on record may be obtained, and a statistical analysis of the peak discharges may be made for the purpose of predicting the peak discharge for a specified return period.

There are several methods of flood frequency analysis.

E. J. Gumbel's [3] extreme-values theory is often used in conjunction with an extreme value probability paper derived by R. W. Powell.[4] This method requires only compilation of the annual maxima for sufficient number of years, from which a curve of discharge vs. frequency is developed.

Hazen [5] log-probability law has also been extensively used by many American hydrologists. Ven Te Chow[6] has developed a table which facilitates the computation, and further

suggested a flexible straight-line fitting of flood data which has the merits of the both the extreme value and log-probability method.

A review of the methods of analysis of flood frequencies was prepared by the subcommittee of the Joint Division Committee on Floods, ASCE[7].

2.2.3 DESIGN FLOOD, BRIDGE ON UNGAGED STREAM

If the bridge is located on a ungaged stream the methods of estimating the peak discharge make use of correlations of physiographic and climatic factors or of a rainfall-runoff relationship. C. F. Izzard,[8] of the U. S. Bureau of Public Roads, has developed a method applicable to the region east of the Rocky Mountains, which gives the design peak discharge as the product of a rainfall factor, a land use and slope factor, a frequency factor and a peak rate of runoff for mixed cover in humid region with frequency of 25 year and rainfall factor of unity. For bridges over streams draining small ungaged watersheds, W. D. Potter (9), of the Bureau of Public Roads, has developed a method of determination of peak rate of runoff from watersheds of less than 25 square miles located East of the 105th Meridian. Several state highway departments - Indiana for example [10] - have developed their own methods. Ven Te Chow [11] has presented a method of peak runoff from small watersheds less than 10 square miles, and its application to Illinois.

The "Rational Method", which gives the discharge as the product of a runoff coefficient, the rainfall intensity, and the drainage area has been popular because of its simplicity for the drainage

design of urban areas and airports. Horner,[12] Potter,[13] and others have questioned the correctness of this formula for flood flow calculations.

A complete listing of empirical formulas for peak discharge determination has been prepared by Ven Te Chow[11].

2.2.4 DESIGN FREQUENCY

The return period established by frequency analysis is the average time interval between events equal to or greater than a given magnitude. The reciprocal of the return period may be thought as the probability that an event occur in any one year. The actual individual return periods may be less than the average. Thus if it is desired to select a design flow which is not likely to occur during the life of the structure, it is necessary to use a return period greater than the estimated useful life of the structure. Linsley et al [14] give the return periods required for specific risk of occurrence within the structure life. Weather the permissible risk of failure should be 0.01 or 0.99 is a matter costs vs. benefits and importance of the structure.

2.2.5 FLOOD DURATION

In certain cases it is required to know the duration of a certain discharge or stage. To obtain this information it is generally needed to obtain the distribution of the runoff with respect to time called the hydrograph. For gaged streams, the flow hydrograph may be derived from the recorded stage graphs. For ungaged streams hydrographs may be developed using the concept of the unit hydrograph. Synthetic hydrographs have been developed for several areas, usually by correlating certain parameters describing the hydrograph, such as the peak discharge, time to peak, etc, to geomorphological and

climatological characteristics of the watersheds. For example, the method developed by [illegible][15] [illegible]plies to the Appalachian Mountain region, that of [illegible] and Schwartz[16] to the North and Middle Atlantic States, that of [illegible][17] to Indiana, etc. Dimensionless unit hydrographs proposed by Williams,[18] Commons,[19] and by the [illegible] Conservation Service[20] give [illegible]

[illegible]

[illegible]

[illegible] analyses [illegible] equations. On th[illegible] has shown that [illegible] streams:

1. The flow [illegible] through the constriction
2. The flow [illegible] critical [illegible]
3. The flow [illegible] throughout the constriction
4. The flow [illegible] from subcritical to supercritical [illegible]

A [illegible] orifice type flow [illegible] bridge opening is submerged. [illegible] subcritical flow throughout, is the most common [illegible]

2.2.7 LIMITATION IN EXTENT [illegible]

The position of maximum backwater or super elevation varies with the amount of constriction and the Froude number. Curves for estimating the distance to the section of maximum backwater have

been prepared by J. N. Bradley[21] of the Bureau of Public Roads. For flow regime No. 2 the backwater upstream of this maximum point follows an M-1 profile[22]. This profile may be calculated by the usual methods of gradually varied flow in open channels.

The extent to which the backwater superelevation is permissible varies with the location. In general submergence of the bridge structure should be avoided for several reasons:

a) the orifice-type flow is hydraulically less efficient than the free surface flow.

b) the accumulation of debris which occurs at the upstream face.

c) the danger of overtopping.

Excessive backwater may provide flooding of a zone upstream of the constriction. The tolerable extent to which backwater is permissible should be determined in each individual case.

2.2.8 LIMITATION IN DURATION OF BACKWATER

The duration during which a certain water elevation will be equalled or exceeded and its frequency may be determined from the flow hydrograph at the site for that frequency, and from the geometry of the constriction. If the backwater superelevation produces the flooding of an area and the interruption of certain services, the duration and frequency of this damage should be estimated. The permissible backwater superelevation, its duration and frequency are determined from economic consideration of cost of damage vs. increased cost of the structure. A useful tool in this analysis is a plot of the peak water surface elevation vs. frequency with and without the structure. Once

the permissible backwater superelevation, its duration, frequency and risk factors are known, the corresponding flood discharges may be evaluated by one of the several methods discussed above, in conjunction with the hydraulic design methods discussed in Section 3.

2.2.9 INDIRECT FLOOD DISCHARGE MEASUREMENT

[illegible]ge contraction, may also have been used as a means of determining flood discharge [illegible]. The fundamental research, which supports the contracted opening method of flood discharge measurement and the values of the discharge coefficients for the several bridge and abutment geometries, have been published by Kindsvater, Carter and Tracy in a circular of the U. S. Geological Survey.[22]

2.3 SITE CHARACTERISTICS

A variety of physical features at the site should be considered in analyzing the hydraulics of a bridge at that site. The hydraulics may be different for the normal flow and for the flood flow.

2.3.1 THE CROSS-SECTION MAY VARY

Natural or man-made features upstream of the bridge may effect the manner in which the flows approach the bridge; the distribution of flood flows on the flood plain may vary as a result of this characteristic.

Similar features downstream may affect the hydraulics. A constriction may result in flood water being ponded at the site.

In addition to a cross-section at the site, several representative samples of cross-sections upstream and downstream should be obtained. A cross-section at a location which is a control for the flow must be obtained.

2.3.2 THE SLOPES OF THE STREAM MAY CHANGE

During normal flows the stream will follow meanders of the low water channel. During floods much of the flow may disregard the meanders; the overall valley features will probably influence the flow patterns.

Channel changes will effect the slope of the stream. Changing the slope could affect the velocity of flow and thus the transport capacity (or scouring ability) of the flow. A channel change downstream of the site may result in degradation of the bridge; upstream of the site may result in aggradation at the bridge.

A profile of a stream bed for a distance to 500 to 1000 ft. upstream and downstream should be obtained. If a channel change is planned, a plan and profile sheet for the old channel and for the new channel should be prepared from which potention troubles may be evaluated.

2.3.3 THE FLOW ANGLE OF APPROACH MAY VARY

For the usual flow, the flow angle of approach will be controlled by the low water channel. At various stages of a flood, the angle or approach may vary until it is controlled by the characteristics or the stream valley.

If scour at the piers and abutments is anticipated, the bridge substructure should be aligned or constructed in a manner which considers the angle of approach which will minimize the scour potential.

Aerial photographs or topographic maps should be studied for possible variations in the angle of approach.

2.3.4 THE CHANNEL ROUGHNESS MAY VARY

An average value for channel roughness will probably decrease as the depth increases. However, if the depth increases so that a large portion of the flow is in the dense portion of vegetation, the value for roughness may increase.

The most adverse results from the varying roughness should be analyzed. As the roughness increases, the depth of flow will increase but the average velocity will decrease.

Aerial or ground photographs and personal visits to the site are methods to evaluate the value for roughness. The U. S. Geological Survey has stereoscopic, colored slides for computed values of roughness; these may be used to compare with conditions at the site.

2.3.5 THE DISCHARGE AND VELOCITY VARY ACROSS THE SECTION

The discharge and velocity will vary at different locations in the section primarily as a result of changes in depth and in roughness. The values for these may be estimated by computing the conveyance at the several sections. The technique has been well developed by the U. S. Geological Survey. This agency makes many bridge site studies for highway departments.

2.3.6 THE BACKWATER WILL VARY DEPENDING ON THE CONSTRICTION GEOMETRY

Backwater at a site is affected by amount of constriction, type of abutments, type of piers, angle of crossing (skew)and eccentricity of crossing.

Methods to compute backwater are contained in "Hydraulics of Bridge Waterways", U. S. Government Printing Office.[21]

2.3.7 SCOUR MAY OCCUR AT SITE

Three forms of scour may occur at a bridge site, the effects of

which may be additive: (1) local scour at a pier as a result of the obstruction to flow, (2) general scour through the site as a result of excessive velocities, and (3) streambed scour or degradation as a result of changes in the stream.

Research has been done at State University of Iowa on evaluation of scour. See Highway Research Board Bulletin No. 4 "Scour Around Bridge Piers and Abutments"[30] and Bulletin No. 8 "Scour at Bridge Crossings".[31]

Embankments constructed of easily scoured material should be protected from scour by (1) vegetation along side of and on the embankment or by (2) prevention of high velocities along embankment.

2.3.8 STREAM LOAD MAY INCREASE DURING FLOOD

More sediment may be in flood waters because of additional scour in stream or removal of bank material. This sediment will tend to reduce scour at site because of the limited capacity of the flow to transport material.

More drift may be in flood waters because of sloughing banks or flood waters covering more wooded flood plain. The drift potential should be considered in determining elevation of bridge.

2.4 CONSTRICTION GEOMETRY

The degree of constriction is a function of several factors related to the bridge and abutment shapes.

2.4.1 BRIDGE SUPERSTRUCTURE GEOMETRY

The elevation of the bridge superstructure relative to the elevation of flood waters has an effect on the constriction.

A low-level bridge (for roads with a small traffic count) may be

inundated for practically all floods. This superstructure should offer the minimum interference to the floods by having a shallow depth; a slab-type construction rather than a beam and flow type is suggested. Wheel guards and railings (if any) should be low and contain large holes to permit easy passage of flood waters. Depth indicators at the bridge ends will advise traffic as to depth of flow over the bridge. The constrictive value must be considered for this type of bridge.

A medium-level bridge will have the low member of the superstructure at an elevation above the stage of a flood of a stated frequency depending upon the overall economy of bridge costs and effects of its loss to traffic. A commonly used value is a 50-year flood. If a flood attains a stage which is in the superstructure, the constriction value is materially increased.

A high-level bridge (probably for navigation) will have superstructure at an elevation which a conceivable flood cannot attain. Thus the bridge superstructure should have no constrictive effects.

The width of the stream at flood stage is a factor in determining the number of spans or bridges to be built to accommodate the flow. The bridge across the low-water channel may be designated as the "main" bridge; the others are "relief" bridges. Some feature, such as a higher portion of ground or a grove of trees, may provide a natural division of the flow to the bridges; otherwise, the designer must choose an arbitrary division. Two types of computations will aid in choosing the division:

(1) the relative conveyance in area upstream of each bridge and

(2) the backwater by each bridge must be at about same elevation

a short distance upstream of the bridges.

The relief bridges must be sized to accommodate the flood flows and not on a localized drainage area, except in some possible cases.

2.4.2 BRIDGE SUBSTRUCTURE GEOMETRY.

The type, shape and orientation of piers and abutments affect the constriction and the scour potential.

The types and shapes of piers and abutments have different coefficients for the computation of backwater. Values are shown in "Hydraulics of Bridge Waterways".[21]

Streamlining the shape of the pier is effective only when the major axis of the pier is approximately parallel to the approaching flow. At sites where the flow may approach at varying angles, consideration should be given to a single shaft cylindrical shaped pier.

2.4.3 SPUR DIKES AT BRIDGE ABUTMENTS

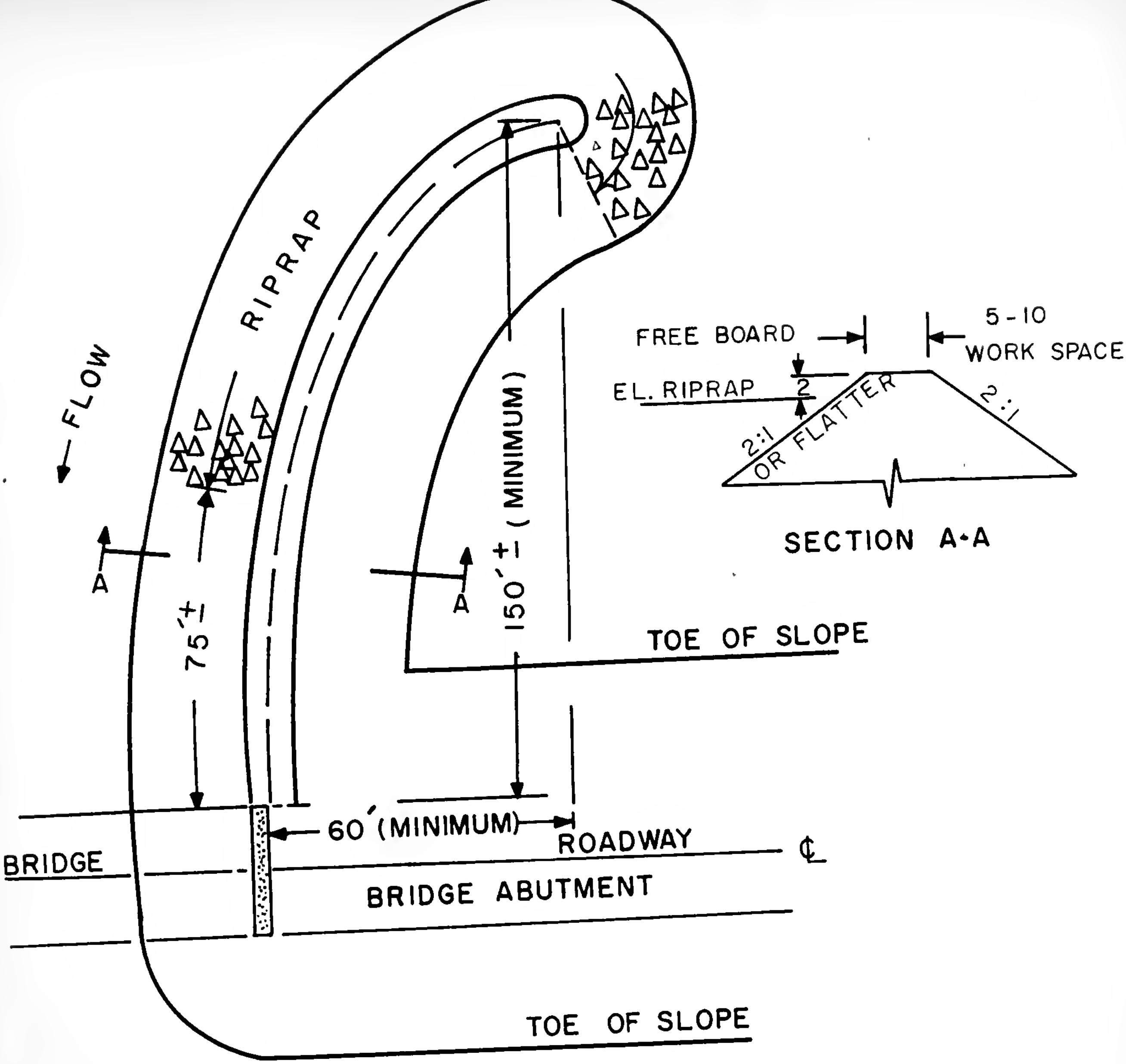
A properly shaped spur dike reduces the constriction and the scour potential at the abutment.

When the flow parallel to the embankment encounters the flow approaching the bridge from a normal direction, an eddy-type action.

Laboratory studies and limited field observation have developed the layout for a spur dike as illustrated on attached sketch. **(Fig. 1)** (See also Article 3.8)

2.4.4 SKEWNESS OF BRIDGE CROSSING

The skewness, or the angle which the bridge and its approaches cross the river and its flood plain, is a factor in the constriction. Values are shown in "Hydraulics of Bridge Waterways."[21]



NOTE 1. SHAPE = ELLIPSE
MAJOR AXIS = 2.5 MINOR AXIS

2. FOR NORMAL CROSSING

FIG. 1 SPUR DIKE LAYOUT

TENTATIVE MAY, 1960

2.4.5 ECCENTRICITY OF BRIDGE CROSSING

The eccentricity, or the offset of the main channel from the middle of the flood plain, affects the constriction. Values are shown in "Hydraulics of Bridge Waterways."[21]

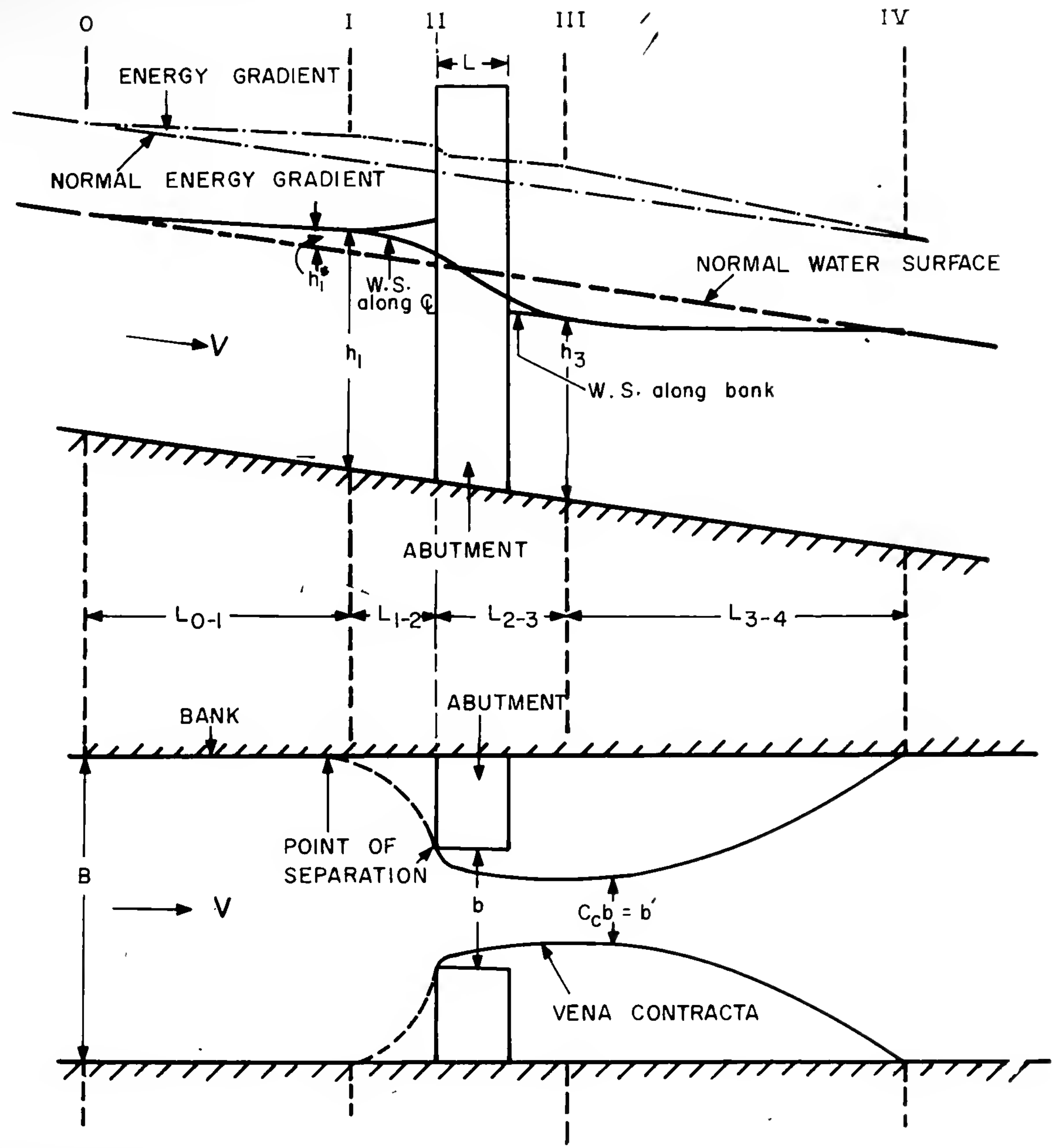
3. HYDRAULIC DESIGN

3.1 BACKWATER FROM OPEN CHANNEL CONSTRICTIONS

A channel transition may be defined as a local change in cross-section which produces a variation in flow from one uniform section to another. An open channel constriction, such as a highway bridge crossing, is an example of a transition of this type. The flow through such constrictions is most often in the tranquil range, and produces gradually varied channel flow far up-and downstream, although at the constriction itself, the flow is of the rapidly varied type.

The effect of the constriction on the surface profile, both upstream and downstream, is conveniently measured with respect to the normal profile, wihch is the water surface in the absence of the constriction. Upstream from the constriction, the backwater profile is of the M-1 variety. In this region, the velocities - and consequently the rate of loss of flow energy - are less than for the normal condition. The backwater effect may extend for a great distance in the upstream direction. At some upstream point, the constricted and the normal profiles coincide.

Near the constriction, the central body of water begins to be accelerated, whereas deceleration occurs along the outer boundaries, and a separation zone is formed in the corners upstream from the constriction. At the constriction, as the flow is accelerated, the average longitudional water-surface profile falls rapidly, and the "live" stream contacts to a width somewhat less than the width of the opening. The spaces between the live stream and the constriction boundaries are occupied by eddying water. Immediately downstream from the constriction, the expansion process begins and continues



DFFINITION SKETCH OF SIMPLE NORMAL CROSSING WITH VERTICAL-WALL ABUTMENT

until the normal regime of flow has been re-established in the full-width channel downstream. At that point, the normal and constricted profiles again coincide. The downstream reach is one of decelerated flow in which the average velocities and energy losses are greater than for the normal case because of the additional turbulent mixing engendered by the expansion process. In the whole backwater reach (the reach between the two points at which the normal and constricted profiles coincide) the total energy loss is the same as that for normal flow.

3.2 ENERGY LOSSES

The effect of the constriction is to cause a re-distribution of the energy of the flow system over the backwater reach. At the constriction, the available energy is greater than for normal flow by an amount required for the increased losses in the downstream reach. The increase in energy is a result of smaller boundary drag loss (as compared with the normal) upstream from the constriction. In the downstream reach, the increased energy losses, also as compared with the normal case, are due primarily to the increased turbulent mixing caused by the diffusion of the live stream as it expands after contraction. These losses are approximately equal to the square of the difference in velocity of the live stream before and after expansion. The difference in velocity, in turn, is a function of discharge, contraction ratio, and the geometry of the constriction, and may be decreased by a decrease in discharge, a smaller contraction ratio, or by an improvement (streamlining) at the abutment and constriction design to more nearly allow the live stream to occupy the full width of the opening. In general, the same statement is applicable to the backwater caused by the constriction.

3.3 MAXIMUM BACKWATER ELEVATION

Although it may be desirable in some cases to predict the complete longitudional profile of the constricted flow throughout the backwater reach, the highway engineer is most usually concerned with the maximum upstream surface change produced by the constriction, and it is to the definition of this latter quantity that the greater part of the backwater studies to date have been devoted. Thus far, two general approaches to the problem are found in the literature. The first is a routing procedure, the second follows from laboratory measurements upon model structures.

The routing procedure assumes that the total energy loss in the downstream backwater reach can be separated into two independant components, (1) the boundary resistance loss, and (2) an enlargement loss. The boundary resistance loss is computed from one of several open channel flow equations derived from studies of uniform flow, and the enlargement loss is computed from the Borda-Carnot equation. Thus, proceeding upstream from a section far enough downstream to include most of the non-uniform flow resulting from the enlargement (the length of this reach is not critical. It should not be so great, however, to cause the enlargement losses to be of a different order of magnitude than the boundary friction losses), the boundary friction losses are computed as for uniform flow, and added to the enlargement losses computed from the Borda equation.

*(Insert statement regarding the coefficient of contraction)

The sum of these losses, added to the energy level at the downstream section, are an approximation of the energy level at the downstream side of the constriction. The energy level, transferred a short

distance above the bridge, is used in conjunction with a diagram of specific energy to find the change in piezometric level at the upstream point due to the constriction, which is the backwater. In spite of the fact that there is no physical justification for the assumption that the energy loss can be divided into independant components, and the use of a uniform flow equation to evaluate the resistance loss in a region of non-uniform flow, this procedure furnishes an approximation to the backwater if no great accuracy is required.

A more direct attack has been made on the backwater problem through laboratory studies on model structures. These studies have had as their objective the measurement and subsequent generalization of the maximum upstream difference between the normal and the constricted longitudional surface profiles, which usually occurs a short distance upstream from the constriction. Considerable attention has been directed, also, to the influence of piers and piling placed in the constricted section as supports for bridge structures in highway crossings. Much of the earlier work on this problem was devoted to the latter subject, however, experimentation was limited, and little of a general nature resulted.

More recent work has provided information more useful to the bridge designer. Practical solutions to the computation of the flow through open-channel constrictions have been furnished by Kindsvater and Carter (46), and by Kindsvater, Carter, and Tracy (23); to the computation of backwater by Tracy and Carter (47), Liu, Bradley, and Plate (48), Biery and Delleur (27), and Bradley (21). Each of the backwater investigations has followed essentially the same basic appraoch, which consisted of the laboratory measurement of the surface level at a selected upstream section before and after the

placement of the constriction in the flow. The difference in surface level was made dimensionless by dividing it by an appropriate flow parameter, and the resulting ratio was related to other variables of the flow or of constriction and channel geometry through correlation techniques used in conjunction with methods of dimensional analysis.

Unless a highway-bridge constriction is located at a gaged site at which a normal stage-discharge relationship has been defined prior to the installation of the structure, prototype verification of the laboratory studies are usually not obtainable. Very little of this type of data are available. Normal stages at bridge crossings are usually synthesized from area-conveyance relationships computed on the basis of a field survey and visual selection of applicable roughness coefficients. This is an approximation, at best, which depends almost entirely upon the skill and experience of the observer. It is generally not satisfactory as a verification procedure in the higher ranges of discharge, particularly where overbank flow is involved. The accumulation of several years of experience with backwater computations at bridge sites have indicated that the backwater may usually be predicted within reasonable limits. The difficulties that have been encounted are common to almost all open-channel flow computations, and stem from the approximations involved in the hydraulic description of channel shape and roughness. An allied problem in prototype channels is the description of the non-uniformity of the flow as it approaches the constriction. It is in these areas that further work on this subject is the most needed.

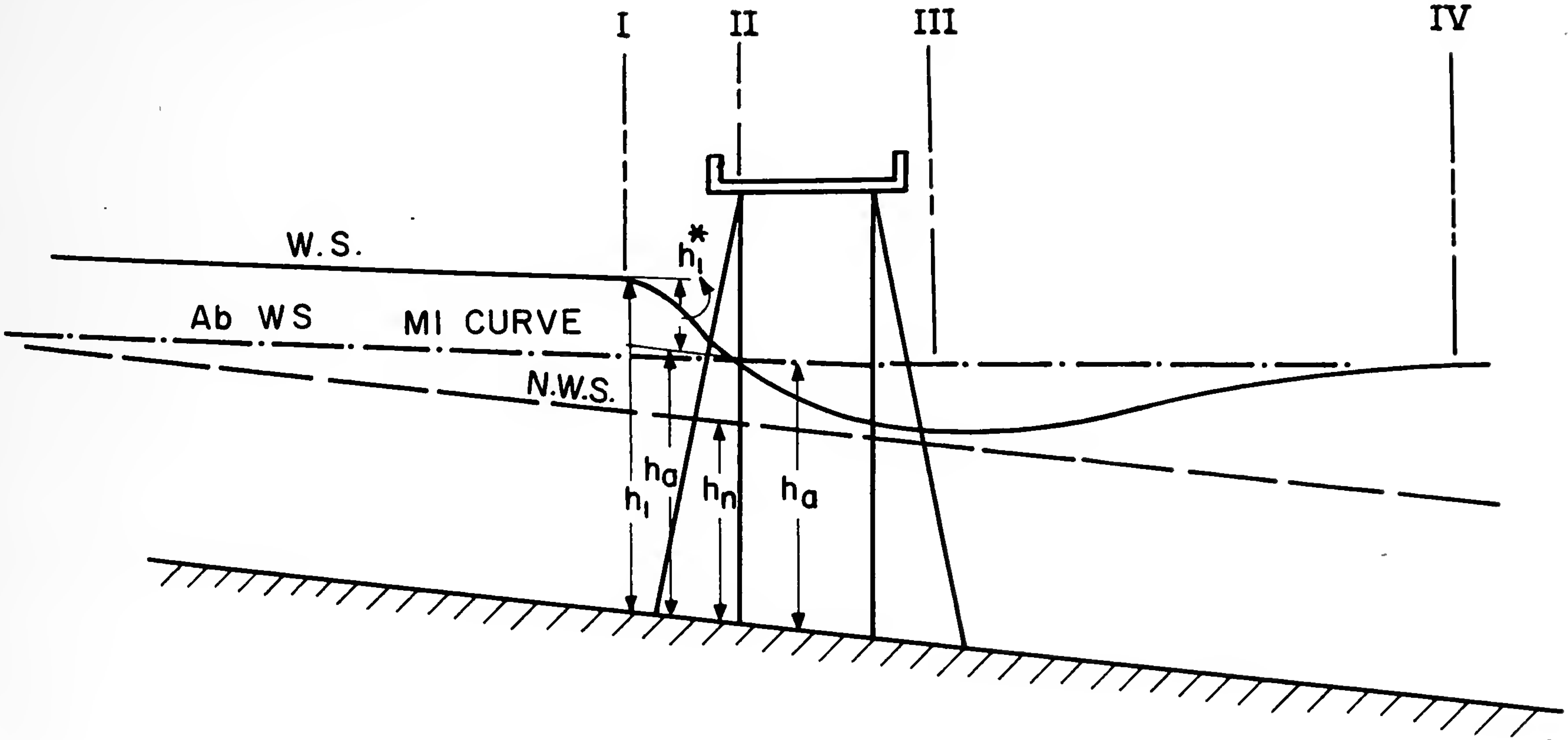
The precedding pages have considered only single-opening constrictions

located in channels on a mild slope. Even for this case, the percentage of channel contraction may be great enough to cause critical flow in the contracted section. When this occurs, the constriction is a control section, and the upstream surface profile is independant of the downstream flow reach, and its determination may proceed from the recognition of this circumstance.

3.4 ABNORMAL STAGE-DISCHARGE CONDITION

Liu, Bradley, and Plate (48) have also considered the condition for which the surface levels in the downstream reach before the placement of the constriction are not controlled by the hydraulic characteristics of that portion of the channel, but are, instead, regulated by a downstream control or transition section. A limited number of laboratory tests were made for this condition, which they called an "abnormal" stage-discharge condition. In the same study, they reported the results of studies involving two constrictions, one being placed downstream from the other. They found that the maximum backwater was created upstream from the upper constriction under a particular combination of opening width and longitudinal spacing.

Davidian, Carrigan, and Shen (49) have considered the case where a single constrictive element may have more than one opening. The division of flow among the openings was related to the area of each opening relative to the total opening area; and on this basis, the boundaries of the flow channel approaching each opening were established. The discharge and backwater characteristics of each opening were than analyzed separately. It was shown that the



DEFINITION SKETCH FOR ABNORMAL STAGE - DISCHARGE CONDITION

backwater relations developed for the single opening constriction are applicable to each opening of a multiple-opening constriction one the boundaries of the separate flow channels are estaolished.

to be added:

discussion of computation of friction loss

verification program in Mississippi

Coefficient of contraction

3.5 VELOCITY DISTRIBUTION IN THE VICINITY OF THE CONSTRICTION

It may be desirable to know the velocity distribution in the vicinity of the constriction in order to estimate possible scour effects and forces on piers. Due to the complexity of the flow field, which is three-dimensional with a free surface, there is no direct way of predicting mathematically the velocity distribution in the vicinity of the constriction. Good approximations to the velocity field may be obtained from model tests. The maximum mean velocity may be roughly approximated by neglecting boundary layer effects in the region of accelerated flow, and by assuming that coefficients of contractions for slots and orifices are applicable to the problem of finding the minimum flow section. Contraction coefficients for slots and orifices are given, for example by Rouse [24]. The maximum mean velocity so obtained may then be compared to the admissible mean volicities in correlation with the type of soil under the bridge. Such a table is given, for example by Jarocki [25].

3.6 REGIONS OF HIGH TURBULENCE AND RECOMMENDATIONS REGARDING POSSIBLE CORRECTIVE MEASURES

The turbulence characteristics of an abruptly expanding flow with expansion half angles of 15°, 30°, 45°, and 90° was studied by Chaturvedi [26] (in the absence of a free surface). These tests simulate somewhat the flow existing in the expansion zone downstream of a bridge constriction although they do not reproduce the free surface effects. It is apparent that the maximum axial turbulence intensity is of the order of 20% and the maximum radial turbulence intensity is of the order of 15%. The points of maximum turbulence intensity occur approximately downstream of the edge of the contraction for a length of about 3 to 4 contraction openings, and

as the flow progresses downstream a more uniform distribution of turbulence intensity is gradually obtained. From these experiments it is apparent that the maximum intensity of turbulence is slightly less for smaller expansion angles, and a uniform distribution of the turbulence intensity is much more rapidly attained for the smaller angles, (in Particular for 15°). It therefore appears that the guideline to follow to decrease the turbulence intensity is to provide wingwalls with a small expansion angle.

3.7 CLEARANCES FOR DEBRIS

The vertical clearance above the elevation of the design flood is usually determined by the drift potential at the bridge site. For average drift conditions, a value of 2 or 3 feet is usually selected. The lateral clearance between piers of the bridge is usually determined by the size of potential drift; large trees will require larger openings to pass through the bridge. Most highway departments have developed standard span lengths for conditions in their State and economical reasons.

3.8 SCOUR AT BRIDGE SITE

The hydraulic design of a highway bridge over a stream subject to flood flows involves the consideration of the possibility of scour, its magnitude, its likely effect on the stability of the structure if not checked, and the most appropriate method of minimizing its effect.

Constriction of the flow of a stream caused by approach embankments or piers results in a rise in the water level immediately upstream of the constriction, and an increase in the stream velocity in the constricted area. Under nor al flow conditions, the higher velocities may not be sufficient to produce significant disturbance

of the material on the stream. In times of flood, the greatly increased velocities may produce severe scouring action, particularly at the bases of abutments and piers, which causes partial or complete collapse of the bridge structure. In the case of a short span bridge, the scour effect increases with the extent of constriction, so that the shorter (and generally the more economical) the bridge structure at a particular site subject to flood scour, the greater is the danger of flood damage due to scour.

Frequently, therefore, the designer's problem involves the determination of the bridge span which, when the cost of scour protection is included, results in the least costly, safe structure. If scour occurs the situation is entirely different from the non-scouring case; the flow pattern is completely different and the essential problem is different. The work at Iowa [30,31] and at Colorado State University [32] both indicate that backwater was so small as not to be measurable with any confidence. The scour relieves the high velocity jet and there are no high velocities to dissipate. There is some disturbance and, therefore there should be some added loss (probably of the order of one velocity head at the most). More important, probably, are the changed flow conditions downstream from the bridge. The flow is confined to the channel and often cannot easily return to the flood plain because of the banks and riverbank vegetation. The dune formation can then be different and the value of the roughness coefficient "n" can be different. Depending upon whether the "n" value is larger or smaller the backwater could be positive or negative. The problem then is scour

and the safety of the bridge and not the maximum backwater as in the case of the rigid bed. The principal difficulty in this approach lies in the lack of reliable data and guidance for predicting the extent of scour under given conditions, and for designing methods of avoiding or reducing its effects. The general problem of sediment entrainment and scour has received considerable research attention over the past thirty years, with, as yet, only limited success. Local scour at bridge sites has been the subject of intensive laboratory and field studies only in the last decade, and there is as yet little guidance available for designers.

3.8.1 POSSIBLE METHODS OF REDUCING SCOUR

One aspect of the bridge scour studies is the economic feasibility of spur dikes, extending upstream from bridge abutments. These dikes are intended to guide the flow into the bridge waterway without the development - near the abutments - of excessively high local velocities. In this way, the scouring action tending to undermine

(cont.)

the abutments is minimised. A knowledge of the most suitable location, form, and dimensions of these dykes, and of the quantitative effects they have in reducing the scouring action would enable the bridge designer to make confident judgments as to their value.

3.6.2 LOCAL SCOUR AT BRIDGE CONTRACTIONS

[illegible] the [illegible] [illegible] to the development of local [illegible] of high local velocity. These generally are in the [illegible] sharp changes in [illegible] wall boundary alignment where the [illegible] separates from the [illegible] so as to produce eddies and zones of high turbulence.

In the case of short span bridges, the extent of channel contraction apparently affects the mean velocity and the local high velocities in the contracted section, and hence the local scour. With bridges of long span, the local velocity and scour effects appear to be less dependent on the overall flow geometry and are treated as purely local phenomena.

3.6.3 DEPTH OF SCOUR

It was not until 1949 that a theoretical approach was attempted in the investigation of scour at bridge sites, when Posey studied briefly, the scour around a pier in the Rocky Mountains Hydraulic Laboratory.(29)

After the disastrous floods in Iowa in the early fifties, the State University of Iowa began investigations into scour around bridge piers and abutments. This work was described by Laursen and Toch in 1956(30), and further work was reported by Laursen in 1958(31) and 1960(35). Some of their conculsions are mentioned below.

Various empirical formulas have been proposed for the depth of

local scour in the [illegible] of long span bridges. Some of these express the depth of scour below the water surface (D_s) as multiple of Lacey's regime depth (D_L) in the contracted section. For example, the Khosla and Inglis formula[32] are of the type

$$D_s = k\,D_L = k \times 0.47\,(Q/f)^{1/3}$$

where Q is the total flow rate, f is Lacey's silt factor, and [illegible] [illegible] [illegible][33] and [illegible][34] [illegible] the bed material. [illegible][35] on the other hand, [illegible] with bed load scour [illegible] during the [illegible] process, [illegible] independent of the [illegible] and flow velocity, [illegible] only on the normal [illegible] the length of the [illegible]. [illegible] that the [illegible] depth of a scour hole [illegible] [illegible] depth of [illegible] the case of [illegible] [illegible] to the [illegible] of the [illegible] with neighboring [illegible] [illegible] of adjacent [illegible].

[illegible] the [illegible] Highway [illegible] measurements of maximum and average depth [illegible] other data relative to [illegible] high water, [illegible] debris, and [illegible] characteristics. A formula [illegible] the average depth of scour to the difference [illegible] sediment load in the approach flow and the transport capacity under the bridge[36].

The more recent laboratory study[32] by Liu, Chang, and Skinner, indicates that the effect of flow velocity on scour may be appreciable, and suggests that Laursen's conclusion to the contrary, holds only for Froude Numbers of less than 0.5 in the unconstricted channel. It concludes that, if the bed load is appreciable, the constriction ratio has no appreciable effect on the depth of scour; but that if there is no bed load, the limiting scour is a function of the constriction ratio. This laboratory investigation yielded experimental curves relating equilibrium and maximum local scour depth to the flow geometry and flow rate (ratio of length of embankment to normal depth, and the normal Froude number, respectively). The authors point out, however, that their results should be used only with caution by designers until prototype verification is obtained.

3.8.4 THE USE OF SPUR DIKES

Spur dikes have been used in a number of cases in the United States to "streamline the flow" through a bridge opening in an attempt to eliminate separation and the accompanying scour. In some cases they are permeable, such as loose rockfill timber cribs, rockfill embankments, and open timber pilings; others, consisting of earth embankments or solid timber sheeting, are impermeable.

In general, separation of the flow at the abutments of abrupt channel constriction results in further contraction of the flow, and hence higher velocities through the constriction. The purpose of the spur dikes is to produce a more uniform velocity distribution and a lower mean velocity, with consequently less liability of scour, through the constriction.

The first study on the effect of spur dikes on the flow pattern in this country was sponsored by the Georgia State Highway

Department. The model spur dikes were made to simulate dikes constructed of timber cribs. It was reported by Carter in 1955, that for spill-through type abutments, a dike of length equal to 0.08B (where B = width of opening) at a distance of 0.08B from the beginning of abutment curvature, and at an angle of 0° to the flow, proved to be the most efficient[37]. No other details were given in the paper.

Some studies were conducted in Sweden in 1957 by Hartzell and Karemyr where dikes were used to align the flow and secure a uniform velocity between the abutments[38]. It appeared that a dike some distance away from the abutment end, and at a 10° angle with the direction of flow, gave best results. However, the tests were inconclusive.

In another Swedish model study of possible erosion at a proposed bridge site[39], it was found that short guide banks extending upstream from the ends of the abutments resulted in appreciable reduction of local scour.

Colorado State University and Lehigh University commenced studies of the effect of spur dikes almost simultaneously early in 1959. The studies at Colorado were conducted in a movable-bed model, while those at Lehigh were in a movable- and fixed-bed model. An elliptically-shaped dike with an axis ratio of 2.5:1 appeared to be most efficient in the Colorado tests. It was also reported[40] that the depth of scour at the abutments is inversely proportional to the length of dike. It was noted that the scour depth is a function of quantity of the flow obstructed or diverted by the embankment. The design criteria were presented for spill-through type abutments, and a tentative guide subject to prototype

confirmation - for determining the length of spur dike was given. In addition, a limited investigation was made for 45° skewed openings. In this part of the study, the depth of scour decreased with increase of length of dike in case of downstream skew, but for the upstream skew, the length of dike did not seem to have any effect on the depth of scour.

The conclusions drawn from the Colorado Studies[41] were that spur dikes are effective in reducing local scour; that their effectiveness depends upon the geometry of the roadway embankments, the flow on the flood plain, and the size of the bridge opening; and that the dike should be curved with its toe alignment at the abutment tangential to the end of the abutment (that is, parallel to the flow in the constriction). With a sloping bank spur dike, this results in the centerline of the dike intersecting the embankment some distance from the end of the embankment.

3.3.5 FIXED-BED SPUR-DIKES MODEL STUDIES AT LEHIGH UNIVERSITY

The objective of the study was to determine the shape and size of dikes necessary for generalized field conditions[42].

In the case of the 90-degree approach (Fig. 2), the spur dikes produced a marked improvement in the uniformity of the velocity across the constriction. The length of dike appeared to be unimportant in reduction of velocities (provided that the length was over a certain minimum length), but the contraction ratio L_o/L_F is important (Fig. 4). In Fig. 4 the change in velocity along the centerline of abutments due to addition of spur dikes is plotted against z/L_o. The average reduction in velocities to about nine-tenths of the original is evident for each of the

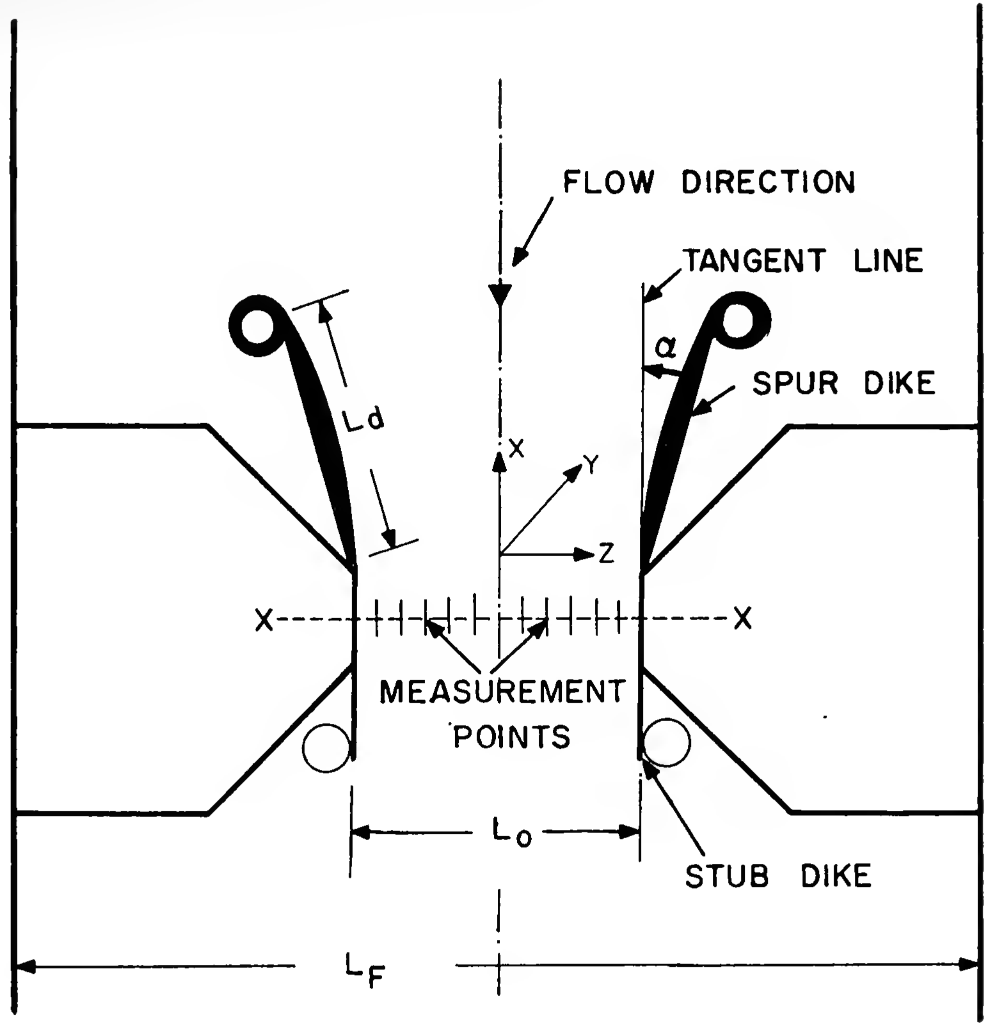


FIG. 2 DEFINITION SKETCH FOR 90° APPROACH FLOW.

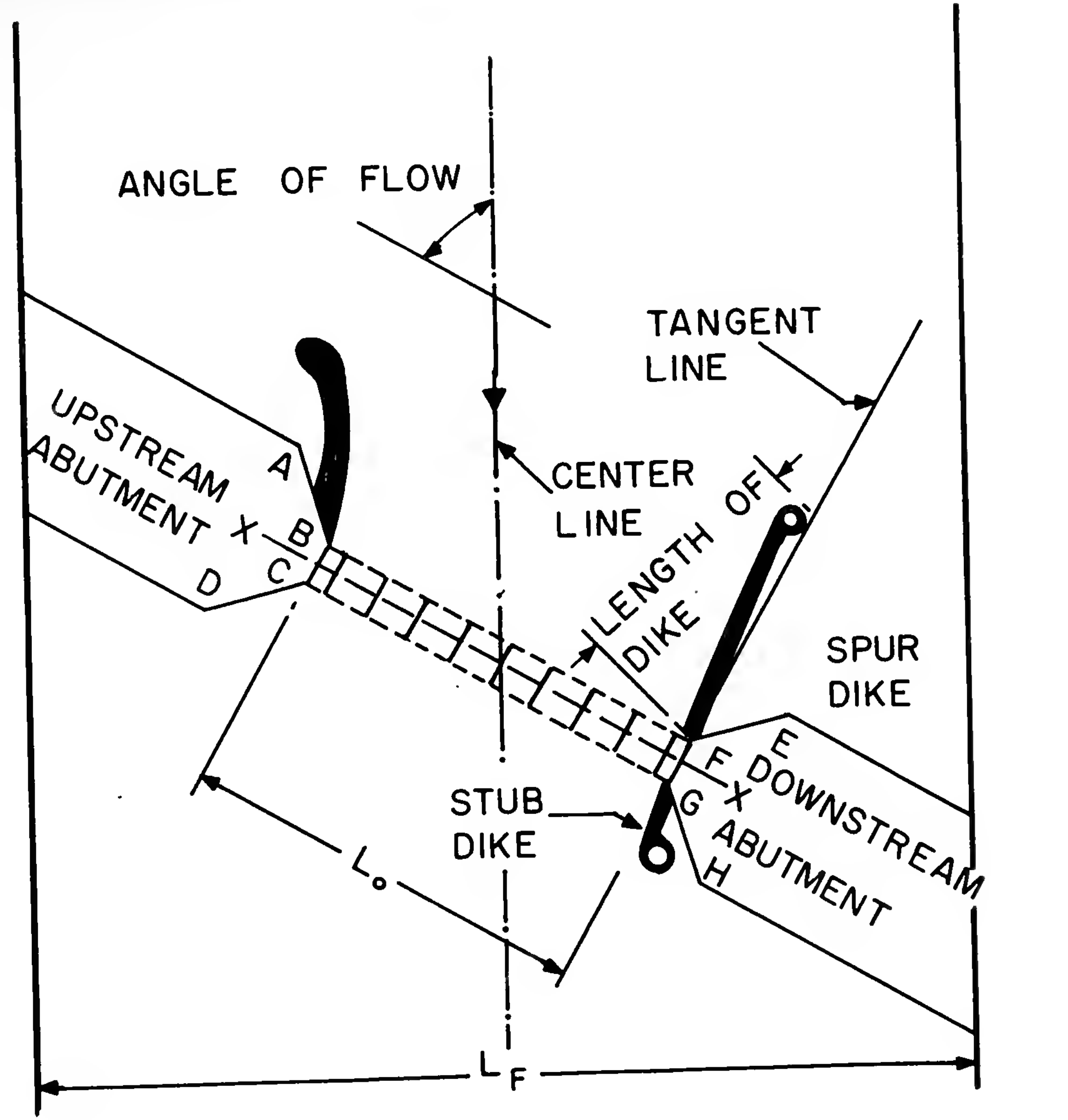


FIG. 3 DEFINITION SKETCH SKEWED ABUTMENT

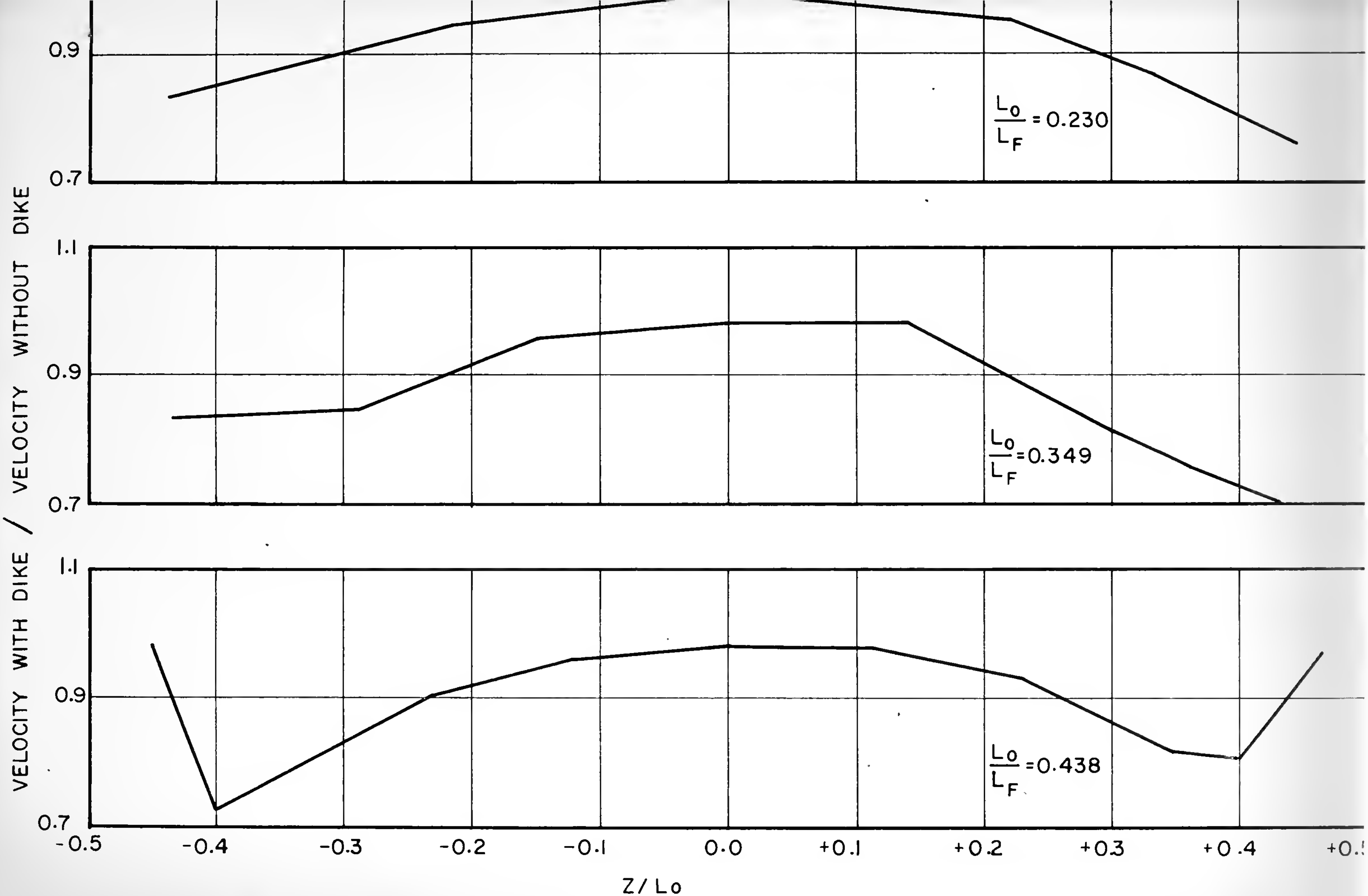


FIG. 4 TYPICAL PATTERNS OF VELOCITY REDUCTION AT CENTER LINE BETWEEN ABUTMENTS BY USING SPUR DIKES 90° APPROACH FLOW

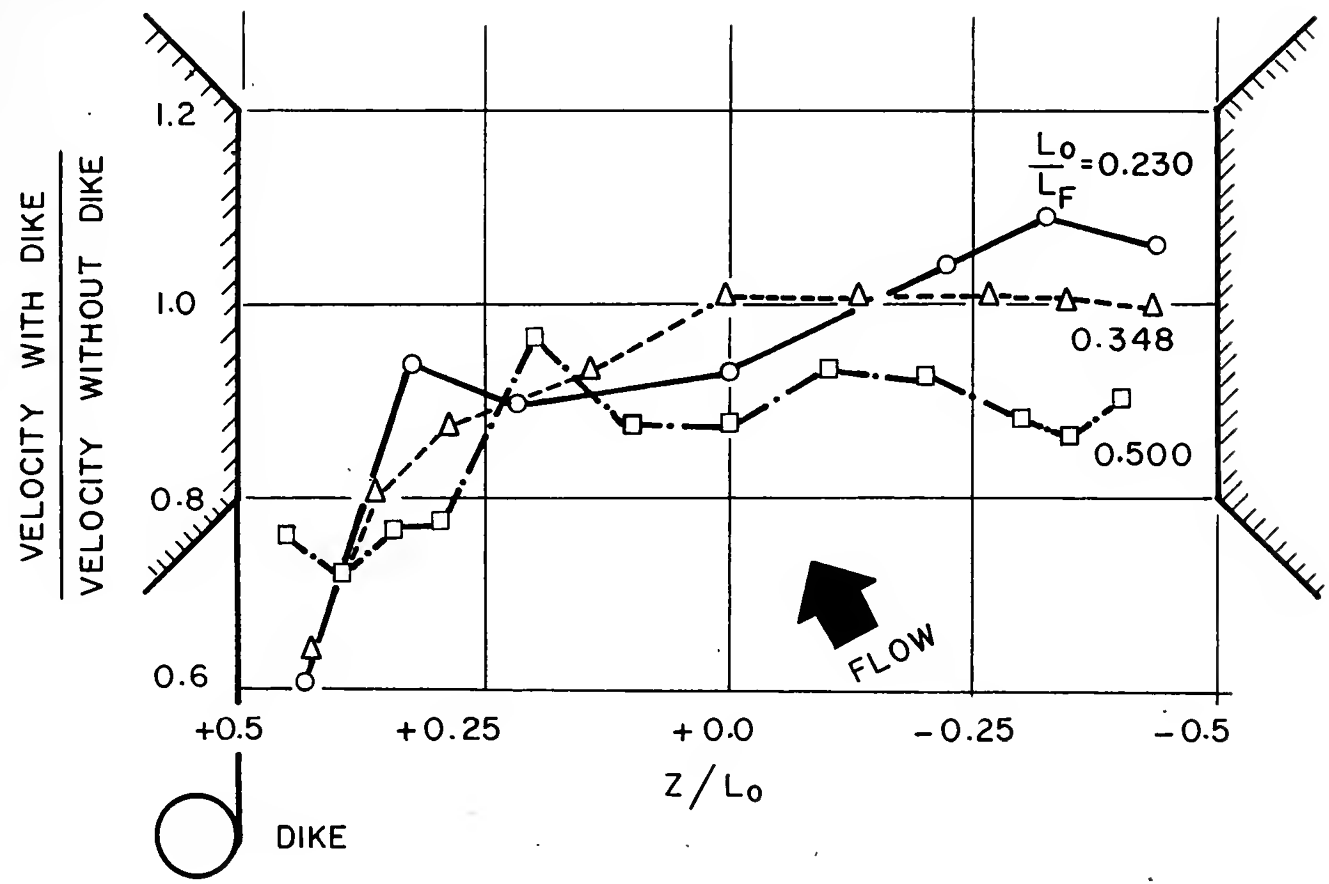


FIG. 5 VELOCITY REDUCTION BY USING SPUR DIKES

60° APPROACH FLOW

contracting ration (L_O/L_F). However, the patterns of reductions are a function of the contracting ratios and it should be noted that the maximum reduction occurs near the abutment, where it is important to prevent high velocities. Thus it may be stated that the average reduction is not as significant as the pattern of reduction.

With the abutments skewed at 60-degrees to the flow, the addition of dikes decreased the velocities along the left-hand abutment to as low as sixty percent of the original. On the right side the velocities increased for 23 percent contraction but decreased for the other contractions (Fig 5). That the greatest contraction should produce the worst condition may be explained by the fact that the fluid flow is deflected toward the right abutment by the dike.

3.8.6 MOVABLE-BED SPUR DIKE MODEL STUDIES AT LEHIGH UNIVERSITY

The movable-bed studies confirmed the predictions based on the fixed-bed investigation that curved spur dikes, in providing a smooth transition for the flow, were extremely effective in reducing scour effects at the abutments. At some points along the abutments deposition occurred where, without dikes, scour would have developed. The following conclusions were reached:

(i) 90-degree approach

(a) Studies of spirally-shaped spur dikes attached to a bridge abutment indicated that such dikes will protect the abutment from damage due to scour. Not only did the dikes significantly reduce maximum scour depth, but they moved the points of deep scour away from the abutments.

(b) The assumptions made in the fixed-bed investigation that uniformity of flow and reduction of eddies produced less scour were verified by the movable bed model study.

(c) In the scour studies of diked-abutments it was found that the mean depth varied as the two-thirds power of the discharge. The same proportionality was reported by Leopold and Wolman for scour between bridges[44].

(ii) 60-degree approach

(a) The condition at a bridge site with skewed abutments is much more severe than with right-angled abutments, and the scour occurs at comparatively low discharges.

3.3.7 PRELIMINARY SPUR DIKE DESIGN RECOMMENDATIONS

(i) 90-degree approach

(a) A curved dike should be used as it eliminates eddying at the head of dike, at the junction of dike and abutment and causes uniform velocities between abutments.

(b) Experimental studies indicate that a spiral shape is suitable to fulfill these requirements. The dike should join the abutment tangentially (fig. 2).

(c) The length of dike itself is not important, provided that it is over a minimum length. The length required to develop a certain shape will usually be greater than the minimum length. The dike should be tangent at the abutment, gradually turning away from the main stream lines to a point having a distance of one-tenth or one-eighth of the width of opening between abutments.

(d) The dike shape should be determined for maximum flow to be expected. This will provide a satisfactory flow for lower discharges.

(e) Shape and length of dike depends upon discharge. In case of high discharge, the shape of the dike should change very gradually. This would cause the dike to be longer than for the case of lower discharge where the transition need not be so gradual.

(f) It should be borne in mind that highest velocities would occur along the dikes in the transition zone and measures should be taken to protect the dike embankment with rip-rap or rock fill.

(ii) 60-degree approach

(a) Comments discussed under 1(a), (d), (e), and (f) apply equally to the 60-degree approach.

(b) Dikes at both abutments are necessary. The most effective shape for the upstream dike is elliptical with axis ratio 2.5:1 (upstream of point B, Fig. 3), and that for the downstream dike is the straight at 5-degree inclination toward the center of the opening (upstream of point F). A stub dike, curved in shape is necessary at the downstream corner (point G) of the downstream abutment.

(c) For the upstream abutment, although a shorter dike is quite sufficient to eliminate scour in front of the abutment, scour at the end of the shorter dike would reach the abutment from behind--consequently a longer elliptical dike is required there.

3.9 SPECIAL GEOMETRIES

The bridge openings discussed in the previous paragraphs refer to deck or girder bridges supported on abutments. The hydraulic characteristics of arch bridges have been studied by Biery and

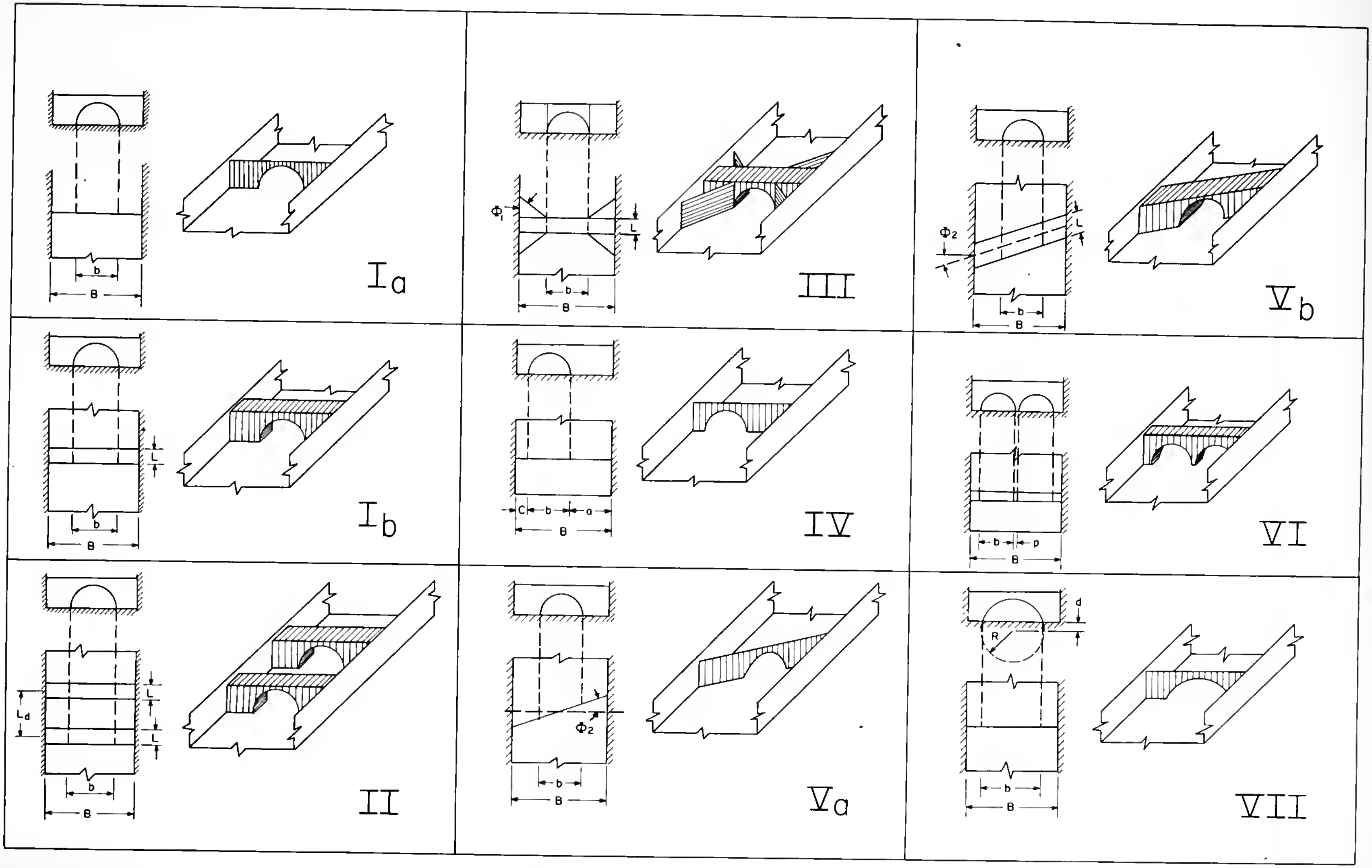


FIGURE 3-14 DEFINITION SKETCHES OF TEST GEOMETRIES

Delleur[27,28]. It appears that the finding for rectangular openings are applicable to arch openings if the channel width ratio (Contraction width ÷ channel width) is replaced by the channel opening ratio (conveyance of constricted opening ÷ conveyance of channel, both computed for the channel normal depth). The channel opening ratio is a function of the stage. Back-water, superelevation and head loss coefficients were evaluated for several geometries, including the effect of wingwalls, skew, eccentricity, dual bridges, two-span bridges and various degrees of submergence.

4. FIELD VERIFICATION

Some field verifications of the hydraulics of bridges have been previously published.

1. Some of the examples in "Hydraulics of Bridge Waterways" [21]
2. "Field Verification of Model Tests on Flow Through Highway Bridges and culverts", State University of Iowa, Studies in Engineering, Bulletin 39.
3. "Some Field Examples of Scour at Bridge Piers and Abutments" BETTER ROADS, August 1954 [50]
4. "Scour Around Bridge Piers and Abutments", Iowa Highway Research Board, Bulletin 4. [30]
5. Bank and Shore Protection in California Highway Practice.
6. "Report on Investigation of Scour at Bridges Caused by Floods of 1955", Highway Research Abstracts, September 1957, Vol. 27, No. 8. [36]

5. ECONOMIC CONSIDERATIONS

Each bridge site will require an economic study based on the characteristics of the specific site. Such a study might be summarized as shown in Fig. 30, page 46 of "Hydraulics of Bridges

Waterways".

This topic is presented in:

1. "Procedure for Determining the Most Economical Design for Bridges and Roadways Crossing Flood Plains", Highway Research Board Bulletin 320,
2. "The Development of Analytic Procedures for Determining Economic Crossings of Streams" in the Highway Research Program of the North Carolina State Highway Department.

The optimum desigh flood for a highway structure is defined by B. W. Gould (45) as that which results in a minimum direct and indirect annual cost. The following annual costs are considered by Gould:

A) the interest on capital

B) the sinking fund for replacement

C) the maintenance cost

D) the risk of destruction of the structure by the minimum flood to destroy or cause substantial damage to the structure

E) the cost due to interruption of the traffic due to the minimum flood to cause traffic interruption

F) the cost of damage caused to crops and property by increased upstream water level due to average annual flood

G) the cost of accident hazards.

6. FIELDS OF RESEARCH, INVESTIGATION AND DISCUSSION

6.1 DATA PRESENTATION

Bridge design authorities frequently have standard data forms to be completed with respect to a bridge design proposal. So far as the hydraulic design is concerned it may be advisable to fix upon

standard data to be provided for adequate hydraulic design. The items listed in Section 2 above might form a starting point and consideration might be given to the standardization of methods of presenting the individual items, particularly those pertaining to site characteristics and constriction geometry. A standard form for presentation of information for hydraulic design should contain data as shown on the following "Hydraulic Report of Proposed Bridge Waterway" which was prepared by Hydraulics Brance, BPR in Washington.

meandering channels [illegible] contemplated on the flood plains and/or the meander halfway. By far the most frequently contemplated change is the construction of a highway crossing the stream

[illegible] [illegible]ear, J. W. "Hydraulics of River Flow under [illegible] Bridges" [illegible] 18, Rept 11, Joint Highway Research Project, June 1964.

(29) Posey, C. J. "Why Bridges Fail in Floods", Civil Engineering February 1949.

(30) Laursen, E. M.; Toch, A. "Scour Around Bridge Piers and Abutments" Iowa Highway Research Board Bulletin, No. 4. 1956

(31) Laursen, E. M. "Scour at Bridge Crossings" Iowa Highway Research Board Bulletin No. 8, 1958

(32) Liu, Chang, and Skinner "The Effect of Bridge Constriction on Scour and Backwater" Publication of C. E. Section, Colorado State University, Report No. CER60HEL22, Feb. 1961.

(33) Blench, T. "Regime Behavior of Canals and Rivers" Butterworths Scientific Pub., London, 1957.

(34) Ahmad, M. "Experiments on Design and Behavior of Spur Dikes" Proc., Minn. Int. Hydr. Conv. 1953.

(35) Laursen, E. M. "Scour at Bridge Crossings" Hydr. Div., Proc., ASCE No. 2369, Feb. 1960.

(36) Moulton, L. K.; Belcher, C.; Butler, B. E. "Report on Investigation of Scour at Bridges Caused by Floods of 1955" Highway Research Abstracts, Vol. 27, No.8, pp. 14-31 Sept. 1957.

(37) Carter, R. W. "Highway Hydraulics" Proceedings of the Fourth Annual Georgia Highway Conference, Feb. 1955.

(38) Hartzell, G.; Karemyr, I. "Anordningar For Minskning Av Risken For Erosion Utanfor Vagbanker (Methods Used for Reduction of Scour at Abutments) Chalmers Tekniska Hogskloa, Sweden 1957.

(39) Reinius, E. "Modellundersokning Av Erosion I Ett Brolage (Model Studies Of Erosion at a Bridge Site) Institute of Hydraulics, Bulletin No. 7, Chalmers Tekniska Hogskola, Sweden 1956

(40) Karaki, S. S. "Hydraulic Model Study of Spur Dikes for Highway Bridge Openings" Civil Engineering Section, Report CER 59 SSK 36 Colorado State University Sept. 1959.

(41) Karaki, S. S. "Laboratory Study of Spur Dikes for Highway Bridge Protection" Paper presented at 39th Meeting of the Highway Research Board, Jan. 1960

(42) Herbich, J. B. "The Effect of Spur Dikes on Flood Flows Through Bridge Constrictions" Paper presented at the National Convention of the American Society of Civil Engineers, Boston Mass. Oct 1960.

(43) Apmann, R.P. "Control of Bridge Scour by Spur Dikes" Fritz Laboratory, Report No 280.17 Lehigh University May 1962.

(44) Leopold, L. B.; Wolman, M. G. Professional Paper 252, U. S. Geological Survey 1960.

(45) Gould, B. W. "Flood Estimation for Highway Structures, Cost Analysis, Optimum Design Flood, Frequency-Flow and Stage Flow Relationships, Interpretation of Frequency" Commonwealth Engineer, October 1, 1956, p. 76-82.

(46) Kindvater, C. E. and Carter, R. W. "Tranquil Flow through Open-Channel Constrictions", Trans. ASCE 120, pp. 955-992, 1955

(47) Tracy, H. J. and Carter, R. W. "Backwater Effects of Open Channel Constriction" Trans. ASCE 120, pp. 993-1018, 1955

(48) Liu, H. K.; Bradley, J. N.; Plate, E. J., "Backwater Effects of Piers and Abutments." Rept. CER 57 HKL 10, Colorado State University 1957

(49) Davidian, J.; Carrigan, P. H. Jr; Shen, J. "Flow through Opening in Width Constriction" U.S.G.S. Water Supply Paper 1369 D, 1962.

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7. ANNOTATED BIBLIOGRAPHY ON HYDRAULICS OF BRIDGES

I. CONSTRICTION AND BACKWATER

BRADLEY, J. N. Use of Backwater in Design of Bridge Waterways [illegible] Roads Vol [illegible] No. 10 Oct. 1959. pg 221-6.

"Investigations carried out by Division of Hydraulic Research Bur. Pub. Roads centered on determination of backwater produced by bridges. Scour at bridge [illegible] scour around piers, the methods for alleviating scour, [illegible] results, design information derived [illegible] application of bridge backwater to waterway design; data presented [illegible] based on experimental backwater studies using hydraulic models and field measurements." [illegible] Eng. Ind. 1960, pg. 160. For a [illegible] detailed discussion see Bradley "Hydraulics of Bridge Waterways"

[illegible] Hydraulics of Bridge Waterways [illegible] Dept. of Commerce Bureau Public Roads, [illegible] Office, 1960

[illegible] waterway design and [illegible] for highway [illegible] engineers [illegible] are given for normal crossing, dual [illegible] skew crossing, eccentric crossing, [illegible] stage discharge and backwater [illegible] The methods of computing the backwater are based almost entirely on model tests conducted at Colorado State University (see Liu, Bradley and Plate)

CHOW, VEN TE. Open Channel Hydraulics. McGraw-Hill Book Company, 1959.

[illegible] the most recent and comprehensive text on hydraulics of open channels. Professor Chow's book contains a chapter on "Flow Through Nonprismatic Channel Sections", (Chapt. 17). A detailed summary (Art. 17-16) with complete set of figures of the work of Kindsvater Carter and Tracy, (reproduced from U.S.G.S. circular 284, 1959) gives all the necessary information to calculate the discharge through constrictions or the backwater ratio due to bridge constrictions

GRAG[illegible], [illegible] Indirect Methods of Discharge Measurement. Proceedings 6th Hydraulics Conference 1955 State University of Iowa Studies in Engineering-Bulletin 36, 1956.

The 1951 flood in the Kansas river basin is taken as an example to discuss the necessity for indirect discharge measurements. The indirect methods are classified in four groups. 1) the slope area, 2) the contracted opening, 3) the flow through culvert, 4) the flow over dam. Each method is discussed briefly.

For a more detailed discussion of indirect discharge measurement at bridge constrictions see Kindsvater, Carter, and Tracy, "Computation of Peak Discharge at Contractions".

[illegible], H. [illegible] Discussion on "[illegible] Channel Constrictions" [illegible] Trans. [illegible], 120, 1003-[illegible] (1955)

By simultaneous use of the momentum equation and the specific energy relation both in dimensionless form, Mr. Henry obtains graphically a theoretical solution for the backwater ratio, thus eliminating the trial and error calculation required by the method of Tracy and Carter (see also Ven Te Chow, Open Channel Hydraulics, example 17-3). The effect of the roughness on the decrease of momentum occasioned by boundary shear in the zone of expansion downstream of the constriction is compared to the roughness effect on the loss of energy on the contracting flow upstream of the constriction.

IZZARD, C. F. Discussion on "Tranquil Flow" by C. E. Kindsvater and R. W. Carter. Trans., ASCE, Vol. 120, pg. 985-89. 1955.

The experiments of Kindsvater and Tracy on "Tranquil Flow through Open-Channel Constrictions", were limited to the case of a horizontal bed. Izzard extends the analysis to the case of sloping channels, and making use of experimental data of Tracy and Carter ("Backwater Effects of Open-Channel Constrictions") he shows that the backwater of the constriction may be expressed approximately as the product of a velocity head (velocity at normal depth in downstream section of constriction) times a coefficient which depends primarily on the contraction ratio of the constriction. Whereas Kindsvater and Tracy were concerned with the problem of estimating the discharge from measurement of water levels in the vicinity of a channel constriction, Izzard is concerned with the reverse problem of estimating the backwater caused by a channel carrying a known flow at normal depth.

IZZARD, C. F. Discussion on "Backwater Effects of Open Channel Constriction" by H. J. Tracy and R. W. Carter. Trans. ASCE Vol. 120 pg. 1008-13. 1955.

The paper by Tracy and Carter is analyzed from the highway engineer viewpoint which is that of calculating expected backwater elevations due to floods of various frequencies. As the accuracy of the flood peak estimates is seldom better than ± 20%, simplifications may be introduced in the backwater calculation. The following simplification is proposed by Izzard. Neglecting minor effects (roughness, and length of constriction) the ratio of the maximum backwater depth upstream of the constriction to the normal depth in the unconstricted channel may be correlated to the contraction ratio and the velocity head in the constricted section. The velocity head in the constricted section is based on the area at normal depth.

IZZARD, C. F. and BRADLEY, J. N. Field verification of model tests on flow through highway bridges and culverts. Proc. 7th Hydr. Conf. Iowa Inst. of Hydr. Res. June 1958, Iowa City, Iowa, State University Iowa 1959, pg. 225-43. AMR 13-4641. Sept. 1960.

The paper reports on comparison of prototype measurements with computed values, derived from model tests for backwater caused by birdges, scour at bridge abutments, and head-discharge characteristics of culverts. Computed and measured values of the drop in water level across the bridge embankment is given for ten sites, two of which are for submerged deck girder. The smallest error is 0.5%, the largest is 13%.

[illegible] at [illegible] of [illegible] the [illegible] the contraction [illegible] length of the [illegible] the discharge coefficient as a function of the primary [illegible] types of [illegible] vertical embankment and vertical abutments; [illegible] embankment [illegible] abutments; [illegible] sloping embankment [illegible] 4) sloping [illegible] with vertical abutments with [illegible] of the discharge [illegible] by [illegible]

[illegible]

[illegible]

[illegible] effect [illegible]

[illegible] computation [illegible] loads, [illegible] due to bridge constrictions are given for single normal crossing [illegible] condition, dual bridge crossing, skew crossing, eccentric crossing [illegible] submerged bridge girders. The water surface profiles, coefficients [illegible] cation of maximum backwater, are presented. This report is perhaps the most comprehensive work on hydraulics of bridge constrictions in the American technical literature

NAGAI, S., On the two-dimensional analyses of suddenly contracted flows. Houille Blanche. Nov., 1950 Pg. 662-73. AMR 7-1849

Position of contraction in suddenly contracted flows is determined by use of the Schwarz-Christoffel theorem. Study is made for flows about a suddenly contracted pipe and about that having a round corner. The position of contraction is approximately given by a determination of the stagnation points [illegible] corner [illegible] vortex [illegible].

[illegible] Effects of [illegible] Constriction [illegible]

[illegible] backwater due to [illegible] channel constriction is given. [illegible] coefficients [illegible] a laboratory investigation [illegible] and influence of channel roughness. The [illegible] through the constriction [illegible] which is the [illegible] backwater [illegible] to be a function of channel roughness [illegible] geometry. [illegible] also the discussion by [illegible].

II. EXPANSION LOSSES

ALBERTSON, M. L., J[illegible], R. A., [illegible], Y. B., ROUSE, H. Diffusion of submerged jets. Trans. ASCE Vol. 115 (1950) pg. [illegible]-697.

This paper deals with the turbulence generated [illegible] air jets issuing from orifices and slots. [illegible] the distribution of the longitudinal velocity [illegible] of particular interest to bridge hydraulics [illegible] under sluice gates.

[illegible] Experimental Determination of [illegible] Head due to [illegible] [illegible] Trans. [illegible] pg. [illegible]

Sudden expansion [illegible] losses

$$[illegible] = [illegible] \frac{[illegible]}{2g} [illegible]$$

[illegible] are the mean [illegible]

[illegible] Les Singularités [illegible] Travaux [illegible] 1945 – Jan. 1946 [illegible]

Problem of turbulence and [illegible] by obstructions in channels and rivers; turbulence [illegible] specific energy; influence of sill at channel bed and [illegible] results of study can be applied [illegible] to channels [illegible] not liable to form deposits. [illegible]

CHATURVEDI, M. C. [illegible] characteristics of [illegible] Jour. of the Hydraulics Division [illegible], Vol. 89 [illegible]

Characteristics of flow for four abrupt expansions with half angles of 15°, 30°, 45° and 90° have been determined by a combination of analytical and experimental mea[illegible] The transformation of mean energy, the rate of production of turbulence, and the rate of dissipation of turbulence energy are determined. The evaluated head-loss results from the air-duct studies are checked by independent measurements in a water-pipe assembly. The kinetic energy [illegible] mean motion, kinetic energy of turbulence, pressure distribution, turbulence production, and turbulence [illegible] presented in the form of their spatial distribution for all expansion angles. Head loss in abrupt expansions is given for different angles of expansion; for 90° (half angle) the head loss coefficient is practically equal to that obtained in the assumption of constant pressure at the inlet section and one-dimensional analysis (p. 80 and Fig. 13). The variation of the head loss with changing expansion ratio is considered (p. 89 and Fig. [illegible]

Consult[illegible] Corp., [illegible]

[illegible] and analysis were [illegible] toward development [illegible] through multiple openings [illegible] approximating [illegible] total discharge among several openings and predicting [illegible] caused by construction [illegible] Engr. Index [illegible]

[illegible]

[illegible]

[illegible] length [illegible] presented in great detail. [illegible]

[illegible]

[illegible] boundary conditions different [illegible] of the flow [illegible] effect and [illegible] instead of [illegible] place of [illegible]

[illegible] coefficient [illegible] the flow under [illegible] opening [illegible] dimensionless [illegible] expansion of the jet [illegible] upper boundary [illegible]

[illegible] Energy Loss [illegible] with [illegible] Enlargements [illegible] Lines. USGS Water [illegible]

[illegible] experimental [illegible] concentric enlargements [illegible] Particular attention [illegible] to the influence of pipe-wall roughness and to methods of computing the energy loss resulting from enlargements. The conventional method of computing the energy loss from the Borda-Carnot equation was found adequate for practical use.

ZELENZIDEV, [illegible] An [illegible] of the [illegible] of Flow in the Region of [illegible] of [illegible] (in [illegible]) [illegible] Nauch. Yuda. [illegible] 6557 [illegible]

[illegible] the [illegible] (boundary) region [illegible] with [illegible] for solutions of [illegible] velocity fields [illegible] of the [illegible] increase [illegible] high of [illegible] of formation [illegible] the [illegible] of section [illegible]

[illegible] construction [illegible] ([illegible]) [illegible] AMR 13-3090

[illegible] special cases [illegible] of hollow [illegible] expression for [illegible] coefficient of contraction in terms of [illegible] ratio of the hole [illegible] flow area of [illegible] and [illegible] the entrance [illegible] some typographical [illegible] well represented by [illegible] investigators. The value of the coefficient of contraction for submerged flow is sometimes calculated [illegible] less [illegible] case of [illegible] into the [illegible] On this basis [illegible] use of his formula for all cases, varying the value of [illegible] given in his previous paper.

KHOMYAK, YA. V., An investigation of the conditions governing the flowing of water in submerged below-bridge currents (in Russian) Trudi-Kharsk. Avtomob.-Dor. In-ta SB. 2, 145-153, 1957 AMR 13-6393 Ref. Zh. Mekh, no 3 1959 Ref. 2649

Results are given of the investigations of the conditions governing the flow of water in the submerged below-bridge openings of rectangular transverse section, together with proposals for an approximate method for the hydraulic calculations, based on data of laboratory experiments carried out by the author. Data of the experiments (curves of the free surface, coefficients of discharge, etc.) are furnished for one of the series: a model of a bridge with openings of 10.1 cm with a length of the buttesses along the flow of 20 cm.

KNAPP, F. H., Flow through orifices, over weirs and under gates in hydraulic structures. Applied hydraulics on a physical basis. Ausfluss, Uberfall and Durchfluss in Wasswebau -- Eine angewadte Hydraulik auf physikalischer Grundlage. Karlsruhe, Verlag G. Braun, 1960 XXiii + 671 pp AMR 14-2514, May 61.

This is a complete treatise of a large part of applied hydraulics, dealing with flow through orifices, nozzles and valves, over free and submerged weirs of different forms, and through variable openings under sluice-gates. Thorough theoretical derivation of formulas of flow is presented. Author considers dynamic pressure distribution caused by curved streamlines and the resulting force effects; these important facts are usually disregarded in computation of weirs and gates. Many recent textbooks of fluid mechanics mention orifices and weirs as devices for flow measurement only. Therefore this book is of great importance for hydraulic designing engineers.

SOUTHWELL and VAISEY. Relaxation Methods applied to Engineering Problems XII, Fluid Motion Characterized by Free Streamlines. Phil. Trans. (A) 240 (1946) 117.

The exact solution (in the sense that no assumption in addition to irrotationality is made) for the sluice gate problem is given.

BLANCHET, C. "Sur Le Probleme Des Remous Et Des Pertes De Charge Produits Par Les Singularites Dans Les Canaux Et Rivieres", La Houille Blanche, Nov. 1945, pp. 39-62.

Water surface profiles due to sills, bridge piers and constrictions are studied making use of the specific energy diagram.

JAROEKI, W. "Hydrologic and Hydraulic Computations of Culverts and Small Bridges" Translated from Polish, available from Office of Technical Services, Dept. of Commerce, 1963; 160 pp.

This extensive treatise covers in detail the methods of computation and the formulas for computing maximum discharges at bridge or culvert sites. A listing of Polish, Russian, South European, German, and American Formulas is given. The section on hydraulic computations covers the water discharge through small openings, the accumulation of water above culverts, the hydraulic computation of small bridge openings and of culvert openings.

HERR, L. A. and SEARCY, J. K. U.S. Bureau of Public Roads "Erosion and Sediment control in Highway Construction", Paper presented at ASCE Water Resources Conference, Milwaukee, Wisconsin, May 13-17, 1962.

The paper states that erosion control must be considered in the design and construction phase rather than [illegible] care of it [illegible] maintenance.

HICKENLOOPER, I. J. [illegible] Changes of a Highway Bridge [illegible] Civil Engineering, [illegible]

The report [illegible] overflow Bridge No [illegible] links [illegible] and [illegible], Illinois. The purpose of the investigation [illegible] of excessive scour downstream of the bridge, and [illegible] remedial solution.

PETER, Y., "Backwater Caused by Bridges", Civil Engineering and Public Works Review, Vol. [illegible], No [illegible], Dec. [illegible], p. [illegible]-1421

An example [illegible] given using the momentum principle. [illegible] with a calculation based on energy principle. Author [illegible]

YARNELL, D. L. "Some Aspects of Flow Around Bends, Bridge Piers and over Highway and Railway Embankments", Proceedings [illegible] Iowa Engineering Society, Des Moines, V.5, Oct. 30, 1930 pp 31-44

This paper reports on the results of experimental investigations done at the University of Iowa in the topics indicated in the title of the paper. The direction of flow and spiral motion in a channel bend were observed. Empirical equations for the discharge over several types of embankments are given.

LIU, H. K.; CHANG, F. M.; SKINNER, M. M. "Effect of Bridge Constriction on Scour and Backwater", Report CER60HKL22, Colorado State University 1961

Scour at Bridge Abutments is the main subject of this report in which the applied ride of the problem is stressed. Scour depth, cross-section, location of maximum scour and scour rate are related to the geometry of the constriction, sediment and flow properties, the maximum backwater and water-surface drop across the embankment are studied. (adapted from author's introduction)

CHANG, F. M.; YEVDJEVICH, V. M. "Analytical Study of Local Scour" Report CER62FMC26, Colorado State University, 1962.

This report is an addendum to the report by Liu, Chang and Skinner, "Effect of Bridge Constriction on Scour and Backwater",; it includes revaiw of additional pertinent literature, discusses further results, describes physical hydrodynamic aspects of loacl scour and suggests further research (adapted from authors foreword).

ABOU-SEIDA, M. M. "Sediment Scour at Structures", Technical Report HEL-4-2, Hydraulic Engineering Laboratory, University of California, 1963

The results of an experimental investigation of local scour around structures due to wave action or a combination of waves and stream flow are given. Circular and square piles, flat plates and hemi-spheres were tested.